Engineering Analysis

4 x 4 Aluminum Column and Post Base

Report F0530.01-122-34

Rendered to:

POLY VINYL COMPANY
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Sheboygan Falls, Wisconsin 53085

Prepared by:

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COMMONWEALTH OF PENNSYLVANIA
REGISTERED PROFESSIONAL
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Scope

Architectural Testing, Inc., an Intertek company, was contracted by Poly Vinyl Company to perform an uplift capacity analysis with anchorage calculations for a 5" x 5" post base and axial load capacity analysis for a 4" x 4" aluminum column. The post base is manufactured with cast 319-F aluminum. The column is manufactured from 6063-T6 aluminum.

Referenced standards utilized in this project include:

Aluminum Design Manual, 2010, The Aluminum Association, 2010.

Metal Curtain Wall Fasteners, American Architectural Manufacturers Association, Report AAMA TIR-A9-1991, 1991.

ANSI/AWC NDS-2012 National Design Specification for Wood Construction, American Wood Council, 2012.

Product Description

Poly Vinyl Company provided drawings of the post base and the aluminum column. The post base is manufactured with 319-F cast aluminum. The column is made from 6063-T6 extruded aluminum. The post base will be anchored to concrete or wood substrates. Powers Tapper+ concrete screws will be used when the post base is anchored to concrete. When the post base is anchored to wood substrate, #14 x 1-1/2" stainless steel wood screws will be used. The aluminum column will be secured to the post base with four (4) #10-12 x 1" 18-8 stainless steel pan head screws. As a base plate, the post base can be anchored to either 2 x 6 Southern Yellow Pine, preservative-treated deck boards or a minimum 4" thick, 3,000 psi, normal weight concrete slab. As a top plate, the post base can be anchored to a Southern Yellow Pine wood header. The columns will be evaluated for 6'-0", 7'-0", 8'-0", 9'-0" and 10'-0" lengths.

Analyses

Allowable Wind Uplift Load

Uplift loads for the column acting on the post base are assumed to be concentric and vertical. Bending of the column on the post base was not considered in the analysis. Maximum allowable uplift load for the post base is based on the lowest values of the post base interaction with the substrate and the post base interaction with the column. Load duration factors for wind loads are utilized. Maximum allowable wind uplift loads are presented in Table 1.

Table 1 Allowable Wind Uplift Load for Post Base

Post Base	Allowable Uplift (lb)	Comments
5" x 5" Casting	432	Limited by shear of #10-12 screws from column to post base.

Notes:

1. At base: post base installed into a minimum 2 x 6 SYP #2 flat plank with four (4) #14 x 1-1/2" countersunk head, stainless steel wood screws.

   or

   Post base installed into a minimum 4" thick f'c = 3,000 psi concrete slab with four (4) Powers Tapper+ 1/4" x 2" countersunk head concrete screws. Minimum embedment of 1-3/4". Minimum edge distance of 2".

2. At top: post base installed into a minimum SYP #2 three-ply header with four (4) #14 x 1-1/2" countersunk head, stainless steel wood screws.

Allowable Axial Compression Column Load

Each column length was analyzed per the Aluminum Design Manual. The compression loads for the columns are assumed to be concentric, acting at the center of the column. Bending of the column was not considered in the analysis. At the base and top, the end condition was assumed to be free rotation with fixed translation. Maximum loads for axial compression are presented in the following table.

Table 2 Allowable Axial Compression Loads

Column Size	Column Length (Feet)				
	6	7	8	9	10
4" x 4" x 0.085"	14,300 lbs.	13,725 lbs.	12,987 lbs.	12,247 lbs.	11,507 lbs.

Summary

Allowable wind uplift loads for the post base connected to a wood base or connected to a concrete base and a two-ply wood header at the top are presented in Table 1. Allowable axial compression loads for the column are presented in Table 2. Loads on the post bases or columns are assumed centered at the column attachment and only vertical loads are considered. Uplift or overturning due to other load types (for example lateral or eccentric loads from the column) must be evaluated by the Engineer of Record for the project.

Attached Drawings

4 Inch Column Base. Part No. 800047. Poly Vinyl Company. 5/06/15. (1 page)

4 Inch Column. Part No. 780097. Poly Vinyl Company. 7/30/15. (1 page)

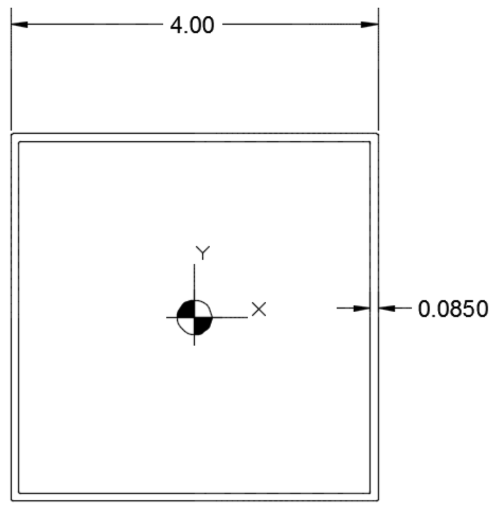
Calculations

Material Properties

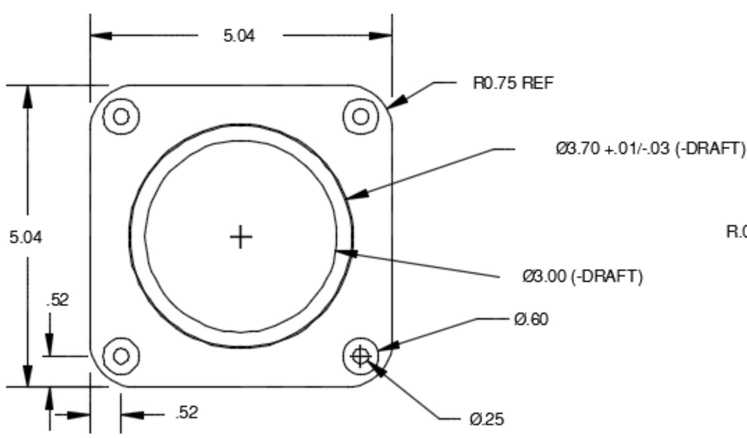
Column Properties
4" x 4" extrusion
Wall thickness = 0.085"
Aluminum 6063-T6: F_y = 25 ksi ; F_u = 30 ksi

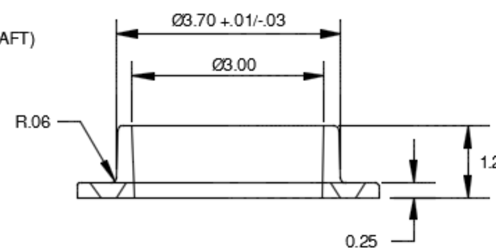
---------- 4" x 4" x 0.085" Column ----------

Area:	1.3311
Bounding box:	X: -2.0000 -- 2.0000 Y: -2.0000 -- 2.0000
Moments of inertia:	X: 3.4018 Y: 3.4018
Radii of gyration:	X: 1.5986 Y: 1.5986



Post Base Properties
Base = 0.25" thick: Collar = 0.35" thick
319-F Cast Aluminum: F_y = 13 ksi ; F_u = 23 ksi





Strength of Column to Post Base Connection

#10-12 x 1" 18-8 Stainless Steel Pan Head Screw

Shear of #10-12 x1" 18-8 Stainless Steel Pan Head Screw

$V_a = (0.75\ F_y/\sqrt{3})(A_R)$. (AAMA TIR A9-14: Ch. 17)
$A_R = (\pi/4)K^2$
$A_R = (\pi/4)(0.126")^2 = 0.0125\ in^2$
$V_a = (0.75)(\ 20{,}000\ psi/\sqrt{3})(\ 0.0125\ in^2) = 108$ lbs.
$V_a = 108$ lbs. x 4 screws = 432 lbs.

Bearing of #10-12 x1" 18-8 Stainless Steel Pan Head Screw (Column thickness controls)

$P_{as} = 2dtFt_u/\Omega$ (AISI Eq. E3.3.1-1)
$P_{as} = (2 \times 0.19" \times 0.085" \times 30{,}000\ psi)/3.0 = 323$ lbs.
$P_{as} = 323$ lbs. x 4 screws = 1,292lbs.

Connection capacity of Post base to Column connection with four (4) #10-12 x 1" 18-8 Stainless Steel Pan Head Screw 432 lbs. Shear of screw controls.

Post Base Connection to 2 x 6 SYP #2 Wood Framing

<u>Post Base Properties</u>
0.25" thick
319-F Cast Aluminum: F_y = 13 ksi ; F_u = 23 ksi

<u>Wood Screw Properties</u>
#14 x 1-1/2" Stainless Steel Wood Screw, countersunk head
Nominal Diameter = 0.242"
Thread Length = 2/3 x (1-1/2") = 1" (NDS Table L3)
Edge Distance = 1.5(0.25") = 0.375" (NDS Table 11.5.A)
End Distance = 4(0.25") = 1" (NDS Table 11.5.B)
Spacing Between screwd = 4(0.25") = 1" (NDS Table 11.5.C)

<u>Wood Base Properties</u>
Minimum 2 x 6 Southern Yellow Pine #2, preservative treated.
G = 0.55
Assume wet service conditions C_M = 0.7

<u>Allowable Tension of #14 Wood Screws (Stainless Steel)</u>

P_{ts}/Ω = 208 lbs. per inch of thread penetration into member: (NDS Table ll.2B)
$P'_{ts} = P_{ts} \times C_D \times C_M$ = 208 lbs. x 1.6 x 0.70 = 233 lbs. per screw
P'_{ts} = 233 lbs. x 4 screws per plate = <u>932 lbs.</u>

<u>Pull-Over of #14 Wood Screw Countersunk Head (Stainless Steel)</u>

$P_{nov} = (0.27 + 1.45t_1/d)d\ t_1F_{TY}/S_F$
P_{nov} = (0.27 + 1.45(0.25/0.242)0.242 x 0.25 x 13,000/3.0
P_{nov} = 463 lbs. per screw
P_{nov} = 463 lbs. x 4 screws per plate = <u>1,852 lbs</u>.

Capacity of Post Base to 2 x 6 SYP #2 Wood Framing is 932 lb. with four (4) #14 x 1-1/2" wood screws. Allowable Tension controls. This capacity is for a wind uplift condition in preservative treated SYP #2 lumber.

Post Base Connection to Concrete

Concrete Screw Properties
Powers Tapper+ 1/4" x 2" concrete screw with minimum 1.75" embedment

Concrete Properties
F_c' = 3,000 psi, un-cracked, normal weight concrete, slab 4" thick minimum. 2" minimum edge distance from plate to slab edge.

Allowable Tension of Powers Tapper+ 1/4" x 2" concrete screw

ΦP_{ts} = 613 lbs. per screw: (See next 3 pages)
ΦP_{ts} = 613 lbs. x 4 screws per plate = 2,452 lbs.
Convert to ASD for Wind: 2,452 lbs. /1.6 = 1,533 lbs.

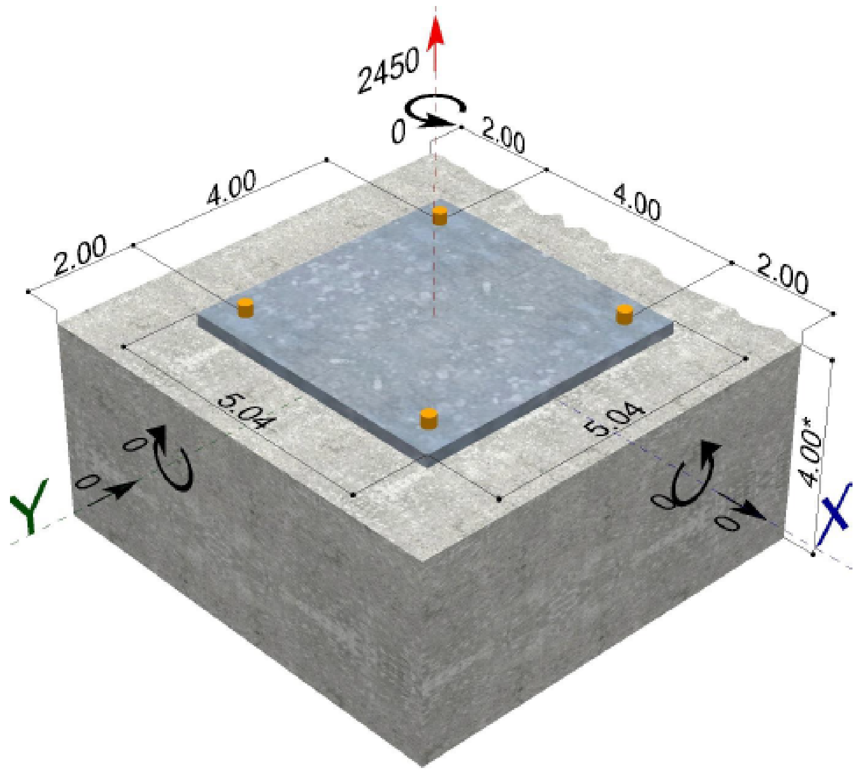
Pull-Over of Powers Tapper+ 1/4" x 2" concrete screw countersunk head

$P_{nov} = (0.27 + 1.45t_1/d)d\ t_1F_{TY}/S_F$
$P_{nov} = (0.27 + 1.45(0.25/0.25)0.25 \times 0.25 \times 13{,}000/3.0$
P_{nov} = 465 lbs. per screw
P_{nov} = 465 lbs. x 4 screws per plate = 1,860 lbs.

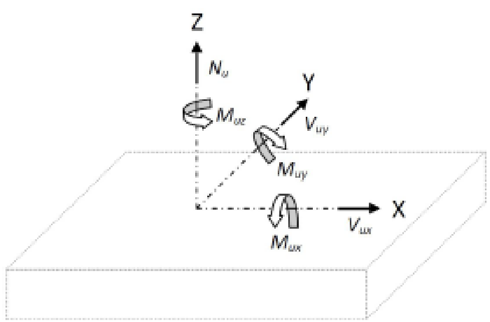
Capacity fo Post Base to 4" thick concrete slab is 1,533 lbs. for wind uplift with four (4) Powers Tapper + 1/4" x 2" screws. Minimum 1.75" embedment for all Tapper+ screws into a minimum 4" thick f'c = 3,000 psi concrete slab. Limited by screw tension.

Post Base Connection to Concrete Base (Continued)

GEOMETRY:



LOAD ACTIONS: [lb], [ft-lb]



Design loads / actions		
N_u	2450	lb
V_{ux}	0	lb
V_{uy}	0	lb
M_{ux}	0	ft-lb
M_{uy}	0	ft-lb
M_{uz}	0	ft-lb

Eccentric profile
e_x = 0.00 inch; e_y = 0.00 inch

Post Base Connection to Concrete Base (Continued)

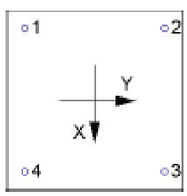
SUMMARY:

Selected anchor: Tapper+
1/4" ; hnom 1-3/4" (45mm), Grade 2
Effective embedment depth: h_{ef} = 1.230 inch
Approval: ESR-3068 (7/1/2013)
Issued: 7/1/2013

Basic principles of Design:

Design method:	ACI 318-11 (Appendix D)							
Concrete:	Normal weight concrete			uncracked concrete		f'_c	= 3000	psi
Load combination:	taken from Section 9.2							
	Factored loads							
Anchor Parameters:	c_{min} = 1.75	inch	s_{min} =	2.00	inch	h_{min} =	3.25	inch
	c_{ac} = 3.00	inch	s_{cr} =	3.69	inch	Anchor Ductility:		No
Reinforcement:	no reinforcement to limit splitting cracks available							
	Tension:	Condition B		Shear:		Condition B		
Stand-off:	not existent							
Seismic Loads:	No							

Resulting anchor forces / load distribution::



Anchor No.	Tension load	Shear load
#1	613 lb	0 lb
#2	613 lb	0 lb
#3	613 lb	0 lb
#4	613 lb	0 lb
Maximum	613 lb	0 lb

Max. concrete compression strain:	0.00	‰
Max. concrete compression stress:	0	psi
Resulting tension force:	613	lb
Resulting compression force:	0	lb

Calculations:	Design proof:	Demand		Capacity		Status	
	Tension load	613	lb	657	lb	0.93 ≤ 1.0	OK
	Shear load	-	-	-	-	-	
	Interaction	-	-	-	-	-	

Anchor plate:

Material:	f_{yk} = 13000 psi		
Length x width:	5.04	inch	x 5.04 inch
Actual plate thickness:	0.250	inch	
Calculated plate thickness:	-	inch	not calculated

Profile: none selected

Post Base Connection to Concrete Base (Continued)

DESIGN PROOF TENSION LOADING:			Reference
Steel strength:			
N_{sa}	= 2680 lb		D.5.1
$\Phi * N_{sa}$	$= \Phi * N_{sa}$		D.5.1.2
	= 0.65 * 2680 lb = 1742 lb		
N_{ua}	= 613 lb		
Design proof:	$N_{ua} / (\Phi * N_{sa})$ = 613 lb / 1742 lb = 0.35 ≤ 1.00		
Concrete Breakout Strength:			
h_{ef}	= 1.230	inch	
k_c	= 24.0		
N_b	$= k_c * f'_c{}^{0.5} * \lambda_a * h_{ef}{}^{1.5}$		D.5.2.2
	= 24.0 * 54.77 * 1.00 * 1.364 = 1793 lb		
A_{Nc0}	= 13.62	inch²	
A_{Nc}	= 13.62	inch²	
$\Psi_{ed,N}$	= 1.000		D.5.2.5
$\Psi_{c,N}$	= 1.00		D.5.2.6
c_{ac}	= 3.00	inch	
$c_{a,min}$	= 2.00	inch	
$\Psi_{cp,N}$	= 0.667		D.5.2.7
$\Phi * N_{cb}$	$= \Phi * (A_{Nc} / A_{Nc0}) * \Psi_{ed,N} * \Psi_{c,N} * \Psi_{cp,N} * N_b$		D.5.2.1
	= 0.65 * (13.62 / 13.62) * 1.000 * 1.00 * 0.667 * 1793 lb		
	= 777	lb	
N_{ua}	= 613	lb	
Design proof:	$N_{ua} / (\Phi * N_{cb})$ = 613 lb / 777 lb = 0.79 ≤ 1.00		
Pullout / Bond strength:			
$N_{p,uncr}$	= 940	lb	D.5.3.2
$\Phi * N_{pn}$	$= \Phi * (f'_c / 2500)$ ^ n $* N_{p,uncr}$		
	= 0.65 * (3000 / 2500) ^ 0.40 * 940 = 657 lb		
N_{ua}	= 613	lb	
Design proof:	$N_{ua} / (\Phi * N_{pn})$ = 613 lb / 657 lb = 0.93 ≤ 1.00		

Fastening ok!

Allowable Axial Compressive Strength of Column

Column Properties
4" x 4" extrusion
Wall thickness = 0.085"
Aluminum 6063-T6: F_y = 25 ksi ; F_u = 30 ksi
r = 1.5986"
area = 1.3311 in^2
K = 1 (base and top end condition: free rotation, fixed translation)

6' Column
Member Buckling
kL/r = (1 x (6' x 12"))/1.5986" = 45.04 < 78
F_c/Ω = 14.2 – 0.074S = 14.2 – 0.074(45.04) = 10.867 ksi (ADM E.3)
$P_n = (F/\Omega)A_g$ = 10.867 ksi x 1.3311 in^2 = 14.465 kips: 14,465 lbs.

Local Buckling
b/t = (4.0 – 2(0.085))/0.085 = 45.06 > 39
F_c/Ω = 484/S = 484/45.06 = 10.741 ksi (ADM B.5.4.2)
$P_n = (F/\Omega)A_g$ = 10.741 ksi x 1.3311 in^2 = 14.3 kips: 14,300 lbs.

Elastic buckling (ADM Section E.5)

$$F_e = \frac{\pi^2 E}{(1.6b/t)^2}$$

$$F_e = \frac{\pi^2 10{,}100}{(1.6 \ x \ 45.06)^2}$$

F_e = 19.177 ksi > 10.867 ksi: Elastic Buckling does not control

Allowable Axial Compressive Strength of 6' Column is 14,300 lbs.

7' Column
Member Buckling
kL/r = (1 x (7' x 12"))/1.5986" = 52.55 < 78
F_c/Ω = 14.2 – 0.074S = 14.2 – 0.074(52.55) = 10.311 ksi (ADM E.3)
$P_n = (F/\Omega)A_g$ = 10.311 ksi x 1.3311 in^2 = 13.725 kips: 13,725 lbs.

Local Buckling
b/t = (4.0 – 2(0.085))/0.085 = 45.06 > 39
F_c/Ω = 484/S = 484/45.06 = 10.741 ksi (ADM B.5.4.2)
$P_n = (F/\Omega)A_g$ = 10.741 ksi x 1.3311 in^2 = 14.3 kips: 14,300 lbs.

F_e = 19.177 ksi > 10.311 ksi: Elastic Buckling does not control

Allowable Axial Compressive Strength of 7' Column is 13,725 lbs.

Allowable Axial Compressive Strength of Column (Continued)

8' Column
Member Buckling
kL/r = (1 x (8' x 12"))/1.5986" = 60.05 < 78
F_c/Ω = 14.2 – 0.074S = 14.2 – 0.074(60.05) = 9.7563 ksi (ADM E.3)
$P_n = (F/\Omega)A_g$ = 9.7563 ksi x 1.3311 in^2 = 12.987 kips: 12,987 lbs.

Local Buckling
b/t = (4.0 – 2(0.085))/0.085 = 45.06 > 39
F_c/Ω = 484/S = 484/45.06 = 10.741 ksi (ADM B.5.4.2)
$P_n = (F/\Omega)A_g$ = 10.741 ksi x 1.3311 in^2 = 14.3 kips: 14,300 lbs.

F_e = 19.177 ksi > 9.7563 ksi: Elastic Buckling does not control

Allowable Axial Compressive Strength of 8' Column is 12,987 lbs.

9' Column
Member Buckling
kL/r = (1 x (9' x 12"))/1.5986" = 67.56 < 78
F_c/Ω = 14.2 – 0.074S = 14.2 – 0.074(67.56) = 9.201 ksi (ADM E.3)
$P_n = (F/\Omega)A_g$ = 9.201 ksi x 1.3311 in^2 = 12.247 kips: 12,247 lbs.

Local Buckling
b/t = (4.0 – 2(0.085))/0.085 = 45.06 > 39
F_c/Ω = 484/S = 484/45.06 = 10.741 ksi (ADM B.5.4.2)
$P_n = (F/\Omega)A_g$ = 10.741 ksi x 1.3311 in^2 = 14.3 kips: 14,300 lbs.

F_e = 19.177 ksi > 9.201 ksi: Elastic Buckling does not control

Allowable Axial Compressive Strength of 9' Column is 12,247 lbs.

Allowable Axial Compressive Strength of Column (Continued)

10' Column
Member Buckling
kL/r = (1 x (10' x 12"))/1.5986" = 75.07 < 78
F_c/Ω = 14.2 – 0.074S = 14.2 – 0.074(75.07) = 8.645 ksi (ADM E.3)
$P_n = (F/\Omega)A_g$ = 8.645 ksi x 1.3311 in^2 = 11.507 kips: 11,507 lbs.

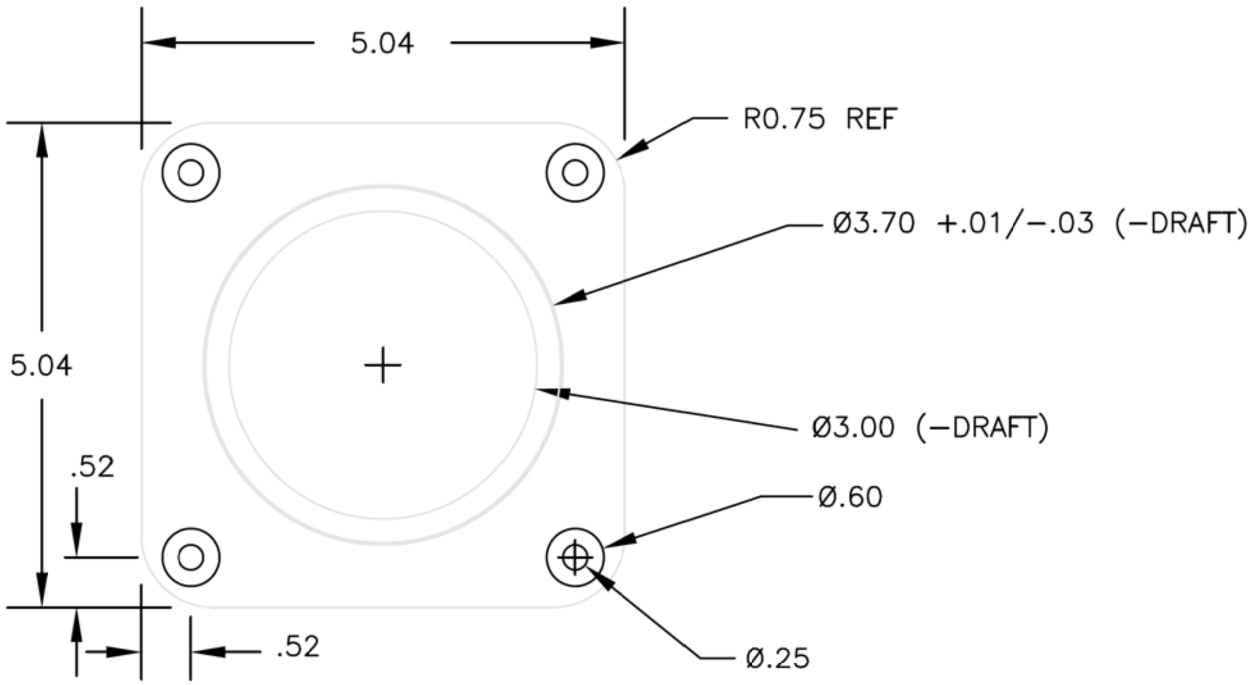
Local Buckling
b/t = (4.0 – 2(0.085))/0.085 = 45.06 > 39
F_c/Ω = 484/S = 484/45.06 = 10.741 ksi (ADM B.5.4.2)
$P_n = (F/\Omega)A_g$ = 10.741 ksi x 1.3311 in^2 = 14.3 kips: 14,300 lbs.

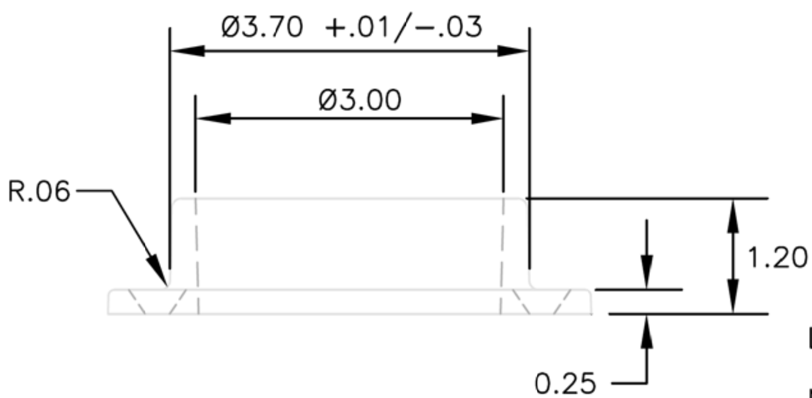
F_e = 19.177 ksi > 8.645 ksi: Elastic Buckling does not control

Allowable Axial Compressive Strength of 10' Column is 11,507 lbs.

Revision Log

Rev. #	Date	Page(s)	Revision(s)
0	08/20/15	N/A	Original report issue





LENGTH-

NOTES:

1 ALUMINUM ALLOY 319
2 FLATTNESS WITHIN 1/32"
3 EDGE FLASHING TO BE MINIMIZED
4 HOLES TO BE CLEAN, NO FLASH
5 PAINT WHITE

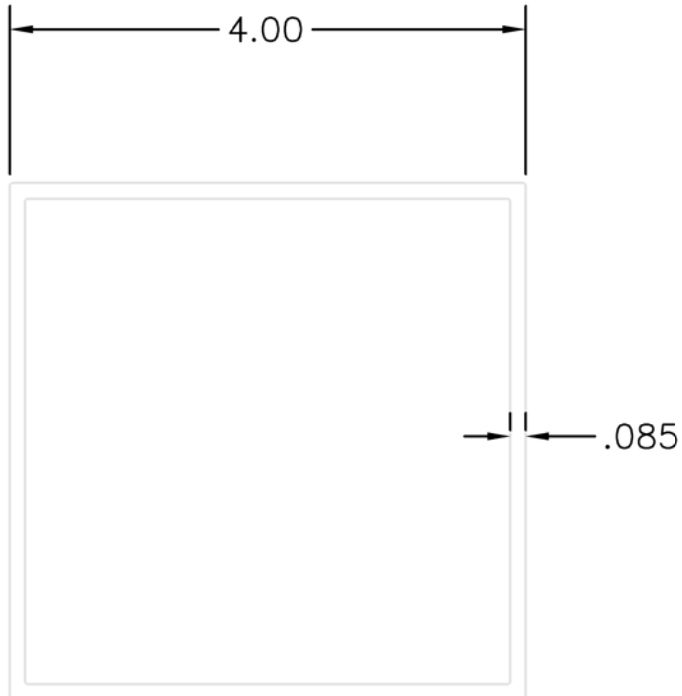
REV	DATE	INTL	EXPLANATION

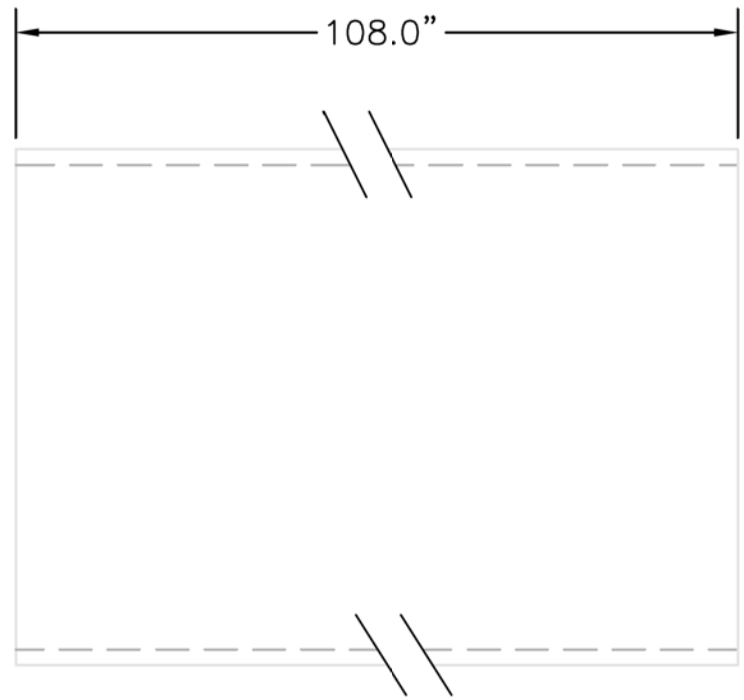
Poly Vinyl Co.
CUSTOM EXTRUSIONS
DRAWER 300 SHEBOYGAN FALLS, WI 53085
PH. (920) 467-4685 FAX. (920) 467-3271

WALL	
AREA-	LOG
FLEXIBLE	REV. -
RIGID	DR. BY RJM
TOLERANCES-	SCALE 1/2
XX ± .030	DATE 05/06/15
XXX ±	DIE
ANGLES	

4" Column Base

PART NO. 800047





780097	No Paint
790097	Black
800097	White
810097	Bronze
820097	Tan
830097	Clay

Poly Vinyl Co.
CUSTOM EXTRUSIONS
DRAWER 300 SHEBOYGAN FALLS, WI 53085
PH. (920) 467-4685 FAX. (920) 467-3271

WALL .085
AREA—
FLEXIBLE
RIGID
LOG
REV.
DR. BY RJM
TOLERANCES—
XX ±
XXX ±
ANGLES
SCALE FULL
DATE 7/30/15
DIE
4 Inch Column
PART NO. 780097